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Specification for the Design of Light Gage Steel Structural Members

American Iron and Steel Institute

SPECIFICATION FOR THE DESIGN OF LIGHT GAGE STEEL STRUCTURAL MEMBERS

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AMERICAN IRON AND STEEL INSTITUTE
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STRUCTURAL MEMBERS

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April 1946

This is the first official edition of the SPECIFICATION FOR THE DESIGN OF LIGHT GAGE STEEL STRUCTURAL MEMBERS issued by the American Iron and Steel Institute.

This Specification has been prepared in response to numerous requests from building officials, engineers, architects and builders for design standards to govern the use of light gage steel members for structural purposes in buildings and similar structures.

In February 1939 the Committee on Building Codes of the Institute initiated a program of structural research to develop the necessary facts. Research work has been carried on intensively and continuously at Cornell University for more than seven years under the direction of Dean S. C. Hollister of the College of Engineering, Professor W. L. Malcolm, Director of the School of Civil Engineering, and Dr. George Winter, Associate Professor of Civil Engineering, in immediate charge of research. The program has included a study of available engineering literature on the subject supplemented by tests on beams, studs or light columns, and deck constructions involving nearly 700 structural specimens. A technical subcommittee under the chairmanship of Milton Male directed the research work and prepared this Design Specification.

In presenting this Specification the Committee clearly recognizes that an initial effort, such as this, cannot pretend to provide final and conclusive answers to all design problems likely to be encountered in practice, but the Committee is of the opinion that this Specification will afford sufficient data for a sound and rational basis upon which to design members formed of sheet and strip steel.

Acknowledgment is given by the Committee to members of the research staff at Cornell University and at other universities who have cooperated in the preparation and analysis of the data upon which this Specification is based, as well as to the many engineers and technicians who have been consulted during the preparation of these design standards.

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EXPLANATORY COMMENTS

The use of sheet and strip steel for structural purposes has become fairly commonplace. The facility with which flat-rolled steel can be formed into simple or complex shapes adaptable to different conditions of service and use has resulted in a wide variety of commercial applications. The variety of shapes which may be produced by cold-forming processes is limited only by the ingenuity of the designer and the economy of the resulting sections for the intended application.

Although the basic design principles applicable to light-gage structural members are the same as those employed for the heavier conventional hot-rolled shapes, it is well to recognize certain supplementary design investigations which are essential in the use of these lighter members. Since they are produced by forming the originally flat-rolled steel into various shapes, cold-formed sections are generally characterized by uniform thickness of all the component parts, in contrast to hot-rolled shapes whose flanges are normally made considerably thicker than the webs. A more important distinction, however, is that the compression elements of cold-formed sections are frequently relatively wide and thin whereas the compression elements of conventional hot-rolled shapes are comparatively thick relative to their width. It is consequently necessary to give consideration in the design and use of cold-formed sections to the prevention of local instability or buckling of the wide, thin compression elements which may comprise part or all of those sections. This feature is adequately provided for in the Specification.

In dealing with thin compression elements, it is important to distinguish between stiffened and unstiffened elements. Theory indicates, and tests have proved, that an *unstiffened* compression element (an element which has *one* flat *unstiffened* edge) will develop sudden local buckling at predictable unit stresses (often below the yield point) referred to the properties of the entire section of which that element is a part. The specification prescribes, therefore, that the full section of *unstiffened* compression elements be used in design but requires a sharp reduction in allowable unit stresses in order to guard against failure by local buckling. The allowable unit stress is directly related to the width-thickness ratio of the element.

In the case of compression elements *stiffened* along *both* edges, however, there is no similar sudden buckling action at a critical stress, but merely a gradual development of buckling waves, gradually reducing the effective section and causing a redistribution of stress intensity as the load is increased. This action starts at relatively low unit stresses in the case of wide elements (high w/t ratios) and at higher stresses in the case of narrower elements. Failure in such *stiffened* compression elements occurs when the maximum stress at the joints between those elements and the other (stiffening) elements of the section approaches yield point intensity. For a section which has *stiffened* compression elements, therefore, the Specification permits design at a maximum allowable unit stress based upon the yield point of the material with an appropriate factor of safety, but requires that the properties of the section (moment of inertia, section modulus, and the like) be based upon a *reduced* or *effective design width,* the value of which is related to the total width-thickness ratio of the element and to the intensity of the compressive stress acting on the element. It is important to note that the application of the concept "effective design width" is limited to *stiffened* compression elements, and that that concept merely requires that a theoretical design width, which is somewhat less than the actual width, be used in computing the properties of the section.

The allowable unit design stresses prescribed in the Specification are intended for application to the nominal thickness of sheet or strip steel forming the structural members, and have been established at conservative values to allow for standard manufacturing tolerances.

In the design of steel compression members used as studs in framing walls, the Specification takes cognizance of the lateral bracing effect of collateral wall sheathing materials and the effect of such bracing on the strength of the steel sections. Similar procedure is common practice in designing wood frame structures. The Specification, however, provides formulae for evaluating the degree of lateral restraint required from these wall materials and their attachments to prevent buckling of the stud, and a method of evaluating the lateral restraint which they are capable of providing.

The Specification consists of (1) design provisions governing the most common shapes and features encountered in the design of light-gage steel structures, which constitute the Specification proper, and (2) four Appendices which are intended to illustrate and clarify the application of some features of the Specification.

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SPECIFICATION FOR THE DESIGN OF LIGHT GAGE STEEL STRUCTURAL MEMBERS

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SECTION 1. GENERAL

1.1—SCOPE

This Specification shall apply to the design of structural members cold formed to shape from sheet or strip steel less than 3/16 inch thick and used for load-carrying purposes in buildings.

Nothing herein is intended to conflict with provisions of the Specifications issued by the American Institute of Steel Construction for the Design, Fabrication, and Erection of Structural Steel for Buildings nor with the Standard Specifications for Steel Joist Construction as adopted by the Steel Joist Institute.

1.2—MATERIAL

Steel shall conform to the Tentative Specifications of the American Society for Testing Materials for Light Gage Structural Quality Flat Rolled Carbon Steel, Serial Designations A245-T and A-246-T, as amended to date, except as otherwise provided herein. The terms C, B, and A when used herein to designate grades of steel shall refer to grades provided by those Tentative ASTM Specifications.

Steel of higher strength than is covered by the above-mentioned ASTM specifications may be used at the unit stresses hereinafter specified for "other" grades of steel provided the design is based upon the minimum properties of those grades of steel as guaranteed by the manufacturer. It is the intent of this Specification to permit the use of high-strength steels of suitable properties for purposes coming within the scope of this Specification, but not to permit the use of ordinary carbon steels at unit stresses higher than those specified in Section 3 for Grade C material.

SECTION 2. DESIGN PROCEDURE

2.1—PROCEDURE

All computations for safe load, stress, deflection and the like shall be in accordance with conventional methods of structural design except as otherwise specified herein. (Explanation of the departures from conventional procedure appears in Explanatory Comments.)

2.2—DEFINITIONS

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) *Stiffened Compression Elements.* The term "stiffened compression elements" shall mean flat compression elements (i.e., plane compression flanges of flexural members and plane webs and flanges of compression members) of which *both* edges parallel to the direction of stress are stiffened by connection to a stiffening means (i.e., web, flange, stiffening lip, intermediate stiffener, or the like) conforming to the requirements of Section 2.3.2.

(b) *Unstiffened Compression Elements.* Any flat element which is stiffened at only *one* edge parallel to the direction of stress shall be considered an "unstiffened" element.

(c) *Flat-Width Ratio.* The flat-width ratio is the ratio, w/t, of the flat width, w, exclusive of edge fillets, of a single flat element to the thickness, t, of such element. In the case of sections such as I-, T-, channel- and Z-shaped sections, the width w is the width of the flat projection of flange from web exclusive of fillets and of any stiffening lip that may be at the outer edge of the flange. In the case of *multiple-web* sections such as inverted U-type or box-shaped sections, the width w is the flat width of flange between adjacent webs, exclusive of fillets.

(d) *Effective Design Width.* Where the flat width, w, of an element is reduced for design purposes, the reduced design width, b, is termed the "effective width," or the "effective design width." (For explanation see Explanatory Comments.) This "effective design width" is determined in accordance with Sections 2.3.1 and 2.3.5. See sketches of "Effective Cross Sections" in Appendix 1.

(e) *Box Type Sections.* See Figure 3 of "Effective Cross Sections" in Appendix 1.

(f) *U-Type Section* (inverted). See Figure 4 of "Effective Cross Sections" in Appendix 1.

(g) *Multiple-Stiffened Elements.* A multiple-stiffened element is one that is stiffened by means of one or more intermediate ribs or stiffeners which are parallel to the direction of stress, and which conform to the requirements of Section 2.3.2, dividing the element into a number of narrower sub-elements each of which shall be considered individually. See Figure 5 of "Effective Cross-Sections" in Appendix 1.

2.3—PROPERTIES OF SECTIONS

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design and shall be based on the full cross-section of the members (or net section where the use of a net section is customary) except where the use of a reduced cross-section, or "effective design width," is required by the provisions of Sections 2.3.1 and 2.3.5 of this Specification.

2.3.1 Properties of Stiffened Compression Elements.

Effective Design Width—In computing the properties of sections of flexural members and in computing the values of "Q" (Section 3.6.1) for compression members, the flat width, w, of any stiffened compression element having a flat-width ratio larger than 25 shall be considered as being reduced for design purposes to an *effective design width,* b, determined in accordance with the following formulae. (The application of these formulae is facilitated by the use of Chart 3 of Appendix 2, or by the use of Table 2.3.1 when the actual stress is 18,000 lb. per sq. in. Chart 3 may be used for any grade of steel and for any stress; Table 2.3.1 is applicable only for Grade C steel and for f = 18,000 lb. per sq. in.)

The curves in Chart 3, and the values in Table 2.3.1, are based on the following formulae: **

For load determinations:

For w/t equal to or less than M,* where

$$M=\frac{(17.5 \times 10^6)+\sqrt{(17.5 \times 10^6)^2-(436 \times 10^6)(7600\sqrt{f}-25f)}}{7600\sqrt{f}-25f},$$

$$\frac{b}{t}=\frac{(17.5 \times 10^6)(w/t-25)}{fM^2}+25$$

For w/t greater than M,*

$$\frac{b}{t}=\frac{7600}{\sqrt{f}}\left[1-\frac{2300}{(w/t)\sqrt{f}}\right]$$

For deflection determinations:

For w/t equal to or less than N,* where

$$N=\frac{(32.2 \times 10^6)+\sqrt{(32.2 \times 10^6)^2-(805 \times 10^6)(10{,}320\sqrt{f}-25f)}}{10{,}320\sqrt{f}-25f},$$

$$\frac{b}{t}=\frac{(32.2 \times 10^6)(w/t-25)}{fN^2}+25$$

For w/t greater than N,*

$$\frac{b}{t}=\frac{10{,}320}{\sqrt{f}}\left[1-\frac{3120}{(w/t)\sqrt{f}}\right]$$

Where w/t = flat width ratio (see Section 2.2)
b = effective design width
f = unit stress in the compression element
(p.s.i.) computed on the basis of the *effective design width.*

That portion of the width considered removed to arrive at the *effective design width* shall be located symmetrically about the center line of the element. See sketches of "Effective Cross Sections" in Appendix 1.

** In using Chart 3 the effective design width should be determined from the actual unit stress in the compression element. In those cases where the compression flange is larger than the tension flange or where the unit compression stress is less than the maximum allowable design stress, the properties of the section can be determined only approximately by using the full design stress. For accurate determinations of properties and effective design width the actual stress must be used since the effective width varies with the unit stress and increases as the latter decreases.

It should be noted that where the w/t ratio is high (about 250 or more) there may be noticeable deformation of the compression element. However, this occurs without detriment to the load-carrying capacity as determined in accordance with this Specification.

* When the actual compression, f, is 18,000 p.s.i., the values for M and N may be obtained from Table 2.3.1.

TABLE 2.3.1

Ratio b/t of Effective Design Width to Thickness of Stiffened Compression Elements, for f = 18,000 lb. per sq. in.

w/t	b/t for		w/t	b/t for	
	Stress	Deflection		Stress	Deflection
25	25.0	25.0	120	48.5	62.8
30	27.5	28.4	140	49.7	64.2
35	30.0	31.5	160	50.5	65.8
40	32.4	34.8	180	51.2	67.0
43.8*	34.5	—	200	51.8	68.0
45	35.1	38.0	225	52.3	69.0
50	37.3	41.3	250	52.8	69.8
52.4**	—	42.8	275	53.1	70.4
60	40.5	47.2	300	53.4	71.0
70	42.8	51.4	350	53.9	71.9
80	44.5	55.3	400	54.2	72.5
90	45.9	57.1	450	54.5	72.9
100	47.0	59.1	500	54.7	73.4

* M value (f = 18,000 lb. per sq. in.) see Section 2.3.1
** N value (f = 18,000 lb. per sq. in.) see Section 2.3.1

2.3.2 Stiffeners for Compression Elements.

2.3.2.1 Minimum Properties and Dimensions of Edge Stiffeners.

In order that a compression element may be considered a "stiffened compression element" it shall be stiffened at each longitudinal edge, running parallel to the direction of stress, by a web, lip, or other stiffening means, having the following minimum moment of inertia:

$$I_{min} = 1.83t^4\sqrt{(w/t)^2 - 144} \quad \text{(See Table 2.3.2.1)}$$

where w/t = flat-width ratio of stiffened element

I = Minimum moment of inertia of stiffener (of any shape) about its own centroidal axis parallel to the stiffened element.

For *intermediate stiffeners* which serve to stiffen compression elements on both sides of the stiffener, the moment of inertia shall be twice that specified for I_{min} above.

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required depth d of such lip may be approximated by using the formula

$$d = 2.8t\sqrt[6]{(w/t)^2 - 144} \qquad \text{(See Table 2.3.2.1)}$$

TABLE 2.3.2.1

Minimum Properties of Stiffening Elements

w/t	I	d	w/t	I	d
12 or less	0	0	20	$29.3t^4$	7.1t
			25	$40.2t^4$	7.8t
13	$9.2t^4$	4.8t	30	$50.4t^4$	8.5t
14	$13.2t^4$	5.4t	40	$69.9t^4$	9.4t
16	$19.4t^4$	6.2t	50	$89.0t^4$	10.2t
18	$24.6t^4$	6.7t	60	$107.6t^4$	10.9t
			Over 60	$1.83t^3w$	$2.8\sqrt[3]{t^2w}$

2.3.2.2 Spacing of Connections.

The longitudinal spacing of rivets or welds connecting a non-integral stiffener to a compression element shall not exceed one-third of the flat width of the stiffened element.

2.3.3 Maximum Allowable Flat-Width Ratios.

Maximum allowable flat-width ratios, w/t, shall be as follows:

(a) Stiffened compression element with one longitudinal edge connected to a web or flange element 60

(b) Compression element stiffened at both longitudinal edges by connection to a web (U-type, box-type sections) or flange 500

(c) Unstiffened compression element 60

Note: The use of unstiffened compression elements having flat-width ratios larger than 30, but not exceeding 60, may result in noticeable distortion of the free edges of such elements without detriment to the ability of the member to support load. Under the limiting stresses prescribed in Section 3.2 hereof such distortion can be expected generally to be in the order of less than 1/16 inch.

(d) *Multiple stiffened elements:* The appropriate maximum allowable flat-width ratios of sub-divisions (a), (b) and (c) of this Section shall apply to the individual sub-elements. The total width of the composite multiple elements shall be limited only by the provisions of sub-division (e) of this Section where applicable.

(e) *Unusually Wide Flanges:* Where a flange is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_{max} = \sqrt{\frac{1{,}800{,}000th}{f_{av}}} \times \sqrt[4]{D}$$

where w_{max} = the width or projection in inches of flange beyond the web fillet for I-beams; or half of the distance between web fillets for box- or U-type beams

t	=	thickness of flange or compression element in inches
h	=	depth of beam in inches
D	=	the amount of curling in percent of the depth, h, (e.g. if 5% of the beam depth is the curling limitation, D = 5 not .05)
f_{av}	=	the *average* stress in the full, unreduced flange width. (Where members are designed by the *effective design width* procedure. the average stress = the maximum stress x (the ratio of the effective design width to the actual width))

See Examples in Appendix 3.

2.3.4 Laterally Unbraced Compression Flanges.

For closed box-type sections the ratio of the laterally unsupported flange length, L, to the gross width of section shall not exceed........ 75

2.3.5 Unusually Short Spans Supporting Concentrated Loads.

Where the span of a beam is less than 30 times the width of its flange projection from the web, and it carries a concentrated load, or loads spaced farther apart than twice the width of flange projection, the effective design width of the tension flange shall be limited as follows:

TABLE 2.3.5

Short, Wide Tension Flanges

Ratio of Effective Design Width to Actual Width

L/w	Ratio	L/w	Ratio
30	1.00	14	.80
25	0.95	12	.75
20	.90	10	.70
18	.86	8	.65

In Table 2.3.5 above:

L = full span for simple spans; or the distance between inflection points for continuous beams; or twice the length of cantilever beams, in inches.

w = width of flange projection beyond the web fillet for I-beam and similar sections or half the flat width of flanges of box- or U-type sections, in inches.

For tension flanges of I-beams stiffened by bent-over lips at the outer edges, w shall be taken as the sum of the flange projection beyond the web plus the depth of the bent-over lip.

SECTION 3. ALLOWABLE DESIGN STRESSES

The maximum allowable unit stresses to be used in design shall be as follows:

3.1—BASIC DESIGN STRESS

Tension on the net section of tension members, and tension and compression, f_b, on the extreme fibers of flexural members shall not exceed the values specified below except as otherwise specifically provided in this Section.

Grade of Steel	Min. Yield Point lb. per sq. in.	f_b (lbs. per sq. in.)
C	33,000	18,000
B	30,000	16,500
A	25,000	13,500
Other	f_b = Minimum specified yield point/1.85	

3.2—COMPRESSION ON UNSTIFFENED ELEMENTS

Compression, f_c, in pounds per square inch, on flat unstiffened elements:

(a) For w/t not greater than 12, $f_c = f_b$

(b) For w/t greater than 12 but not over 30:

$$f_c = [1.67\ f_b - 5430] - (1/18)\ (f_b - 8150)\ w/t$$

where w/t = flat-width ratio, (See Section 2.2)

(Values in accordance with the above formula are given in Table 3.2(b) below. See Examples in Appendix 3.)

TABLE 3.2(b)
Allowable Design Stresses
ASTM A245-T and A246-T Grades of Steel
For w/t Ratios From 12 to 30

w/t	Grade C	Grade B	Grade A
12	18,000	16,500	13,500
14	16,910	15,580	12,910
16	15,810	14,650	12,310
18	14,720	13,720	11,720
20	13,630	12,790	11,130
22	12,530	11,860	10,530
24	11,440	10,940	9,940
26	10,340	10,010	9,340
28	9,250	9,080	8,750
30	8,150	8,150	8,150

(c) For w/t over 30 but not over 60:

$$f_c = 12{,}600 - 148.5\ (w/t)$$

(Values in accordance with the above formula are given in Table 3.2(c) below. See Examples in Appendix 3.)

TABLE 3.2(c)
Allowable Design Stresses**
For w/t Ratios From 30 to 60 (All Grades of Steel)

Ratio w/t	f	Ratio w/t	f	Ratio w/t	f
30	8150	40	6660	50	5180
32	7850	42	6360	52	4880
34	7550	44	6070	54	4580
36	7250	46	5770	56	4280
38	6960	48	5470	58	3990
				60	3690

** Flanges or compression elements having ratios of w/t larger than 30 may show noticeable distortion of the free edges under maximum working stress without detriment to the ability of the member to support load. For the stresses and range of ratios provided in the formula above and in Table 3.2(c) the distortion generally will be of the order of less than 1/16 inch.

For ratios beyond w/t = 60 distortion of the flanges is likely to be so pronounced as to render the section structurally undesirable unless load and stress are limited to such a degree as to render such use uneconomical.

3.3—LATERALLY UNBRACED COMPRESSION FLANGES

Maximum compression, f'_c, in pounds per square inch, on extreme fibers of laterally unsupported compression flanges of straight I-shaped flexural members, (not including multiple-web deck, U- and closed box-type members and curved or arch members), shall not exceed the allowable stress as specified in Section 3.1 or 3.2 nor the following maximum stresses:

$$f'_c = \frac{250,000,000}{(L/r_y)^2}$$

Where L is the unbraced length of the member, and r_y is the radius of gyration of the entire section of the member about its gravity axis parallel to the web; both in inch units.

See Examples in Appendix 3.

3.4—ALLOWABLE WEB SHEAR

The maximum average shear stress, v, in pounds per square inch, on the gross area of a flat web shall not exceed:

$$v = \frac{64,000,000}{(h/t)^2} \text{ with a maximum of } 2/3\ f_b.$$

In the above formula, $t =$ web thickness, $h =$ clear distance between flanges, and $f_b =$ basic working stress as specified in Section 3.1.

Where the web consists of two or more sheets, each sheet shall be considered as a separate member carrying its share of the stress. If, in such cases, the sheets are joined together by continuous welds or by rows of spot welds parallel to the flanges, "h" shall be the vertical distance between the rows of welds or between a row of welds and the flange, whichever is the greater, (rather than the distance between the flanges) provided the longitudinal spacing of welds along each row of welds does not exceed h/3.

Tabular values are given in Table 3.4, for Grade C Steel.

TABLE 3.4
Maximum Allowable Web Shear on Flat Webs
Grade C Steel

$$v = \frac{64,000,000}{(h/t)^2} \text{ with maximum } 2/3\ f_b = 12,000 \text{ lb. per sq. in.}$$

h/t	v	h/t	v	h/t	v
73	12,000	110	5,290	150	2,840
80	10,000	120	4,440	160	2,500
90	7,900	130	3,790	170	2,210
100	6,400	140	3,270		

See Examples in Appendix 3.

3.5—WEB CRIPPLING OF BEAMS

To avoid crippling of flat webs of beams:

(A) Concentrated load located anywhere on the span, or reaction of continuous supports shall not exceed P_{max} in the formula,

(a) .... $P_{max} = t^2 f_b (11.1 + 2.41 \sqrt{B/t})$

For any given load or reaction, P, as defined above, the minimum length of bearing, B_{min}, shall be,

(b) .... $^*B_{min} = t\left[\dfrac{P}{2.41t^2 f_b} - 4.62\right]^2$

(B) Concentrated loads on the outer ends of cantilevers, or simple end reactions of beams, shall not exceed P_{max} in the formula,

(c) .... $P_{max} = t^2 f_b (7.4 + 0.93 \sqrt{B/t})$

For any given load or reaction, P, as defined above, the minimum length of bearing, B, shall be,

(d) .... $^*B_{min} = t\left[\dfrac{P}{0.93t^2 f_b} - 8.00\right]^2$

In these formulae

P = concentrated load, or reaction, in pounds.

t = web thickness, in inches.

B = length of bearing, in inches.

f_b = basic design stress, in pounds per square inch (Section 3.1).

See Examples in Appendix 3.

Where a concentrated load is applied to the top flange of a beam at the support (such as a stud resting on a beam over the support), the required length of bearing to provide for the load on the top flange and for the reaction on the bottom flange shall be determined independently for the upper and lower flanges by the above formulae.

Where the web of a beam consists of two or more sheets, each sheet shall be considered as a separate member carrying its share of the load or reaction.

3.6—AXIALLY LOADED COMPRESSION MEMBERS

3.6.1 Unit Stress**

The allowable unit stress, P/A, for axially loaded compression members shall be:

For Grade C Steel†

L/r less than $132/\sqrt{Q}$: $P/A = 15300Q - 0.437Q^2(L/r)^2$

L/r greater than $132/\sqrt{Q}$: $P/A = \dfrac{134{,}000{,}000}{(L/r)^2}$

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* In formulae (b) and (d) should the first term within the brackets be smaller than the second, the smallest length of bearing which is practicable will be safe and sufficient, as far as web crippling is concerned, for the given load.

** For continuous compression chords of trusses with rigid welded connections at panel points, tests in accordance with Section 6 may show higher allowable load-carrying capacity than calculated by these formulae.

† P/A values for Grade C Steel are given in Chart 4 of Appendix 2.

For Other Grades of Steel

L/r less than $\dfrac{24{,}000}{\sqrt{f_y}\ \sqrt{Q}}$: $\quad \dfrac{P}{A} = 0.464Qf_y - \dfrac{4.01Q^2(f_y)^2(L/r)^2}{10{,}000{,}000{,}000}$

L/r greater than $\dfrac{24{,}000}{\sqrt{f_y}\ \sqrt{Q}}$: $\quad \dfrac{P}{A} = \dfrac{134{,}000{,}000}{(L/r)^2}$

In the above formulae,

P = total allowable load, pounds;
A = full, unreduced cross-sectional area of the member;
L = unsupported length of member, inches;
r = radius of gyration of full, unreduced cross-section, inches;
f_y = yield point of steel, lb. per sq. in.; and
Q = a factor determined as follows:

(a) For members composed entirely of *stiffened* elements, "Q" is the ratio between the effective design area, as determined from the effective design widths of such elements, and the full or gross area of the cross-section. The effective design area used in determining Q is to be based upon the the basic design stress f_b as defined in Section 3.1.

(b) For members composed entirely of *unstiffened* elements, "Q" is the ratio between the allowable compression stress f_c for the weakest element of the cross-section (the element having the largest flat-width ratio) and the basic design stress f_b; where f_c is as defined in Section 3.2, and f_b is as defined in Section 3.1.

(c) For members composed of *both stiffened and unstiffened* elements the factor "Q" is to be the product of a stress factor Q_s computed as outlined in (b) above and an area factor Q_a computed as outlined in (a) above, except that the stress upon which Q_a is to be based shall be that value of the unit stress f_c which is used in computing Q_s; and the effective area to be used in computing Q_a shall include the full area of all unstiffened elements. (See Example 7, Appendix 3.)

See Chart 4 of Appendix 2 for values of P/A for Grade C Steel.

3.6.2 Maximum Slenderness Ratio.

The maximum allowable ratio L/r of unsupported length, L, to radius of gyration, r, of compression members shall be as follows:

(a) Columns, and other primary compression members, except as provided otherwise in this Section 120
(b) Load-bearing studs 160
(c) Secondary members 200
(d) During construction 300

If members which are temporarily unbraced during construction are to act as permanent load-carrying members in the completed structure they must be so braced prior to completion of the structure as to reduce the L/r ratio to a value not exceeding that given in (a), (b), or (c) above, whichever may apply.

3.7—COMBINED AXIAL AND BENDING STRESSES

For members subject to both axial compression and bending stresses, the member shall be so proportioned that the quantity—

$\dfrac{f_a}{F_a} + \dfrac{f'_b}{F_b}$ shall not exceed unity, where

F_a = maximum axial unit stress in compression that is permitted by this Specification where axial stress only exists. (Section 3.6.1)

F_b = maximum bending unit stress in compression that is permitted by this Specification where bending stress only exists. (Section 3.1 and 3.2)

f_a = axial unit stress = axial load divided by full cross-sectional area of member.

f'_b = bending unit stress = bending moment divided by section modulus of member, noting that for members having stiffened compression elements the section modulus shall be based upon the effective design widths of such elements.

SECTION 4. CONNECTIONS

4.1—GENERAL

Connections shall be designed to transmit the maximum stress in the connected member with proper regard for eccentricity. In the case of members subject to reversal of stress the connection shall be proportioned for the sum of the stresses.

4.2—WELDS

4.2.1 Fusion Welds.

For all grades of steel, fusion welds shall be proportioned so that the unit stresses therein shall not exceed 11,300 lbs. per square inch in shear, or 13,000 lbs. per square inch in tension. Stresses due to eccentricity of loading, if any, shall be combined with the primary stresses; and the combined unit stresses shall not exceed the values given above.

4.2.2 Resistance Welds.

In plates or sheets joined by spot welding, the design strength per spot shall be as follows:

Thickness of Thinnest Outside Sheet, Inches	Design Strength per Spot, Pounds
.010	50
.020	125
.030	225
.040	350
.050	525
.060	725
.080	1075
.094	1375
.109	1650
.125	2000
.155	2700
.185	3300

(The above values are those recommended by the American Welding Society in its Recommended Practice for the Spot Welding of Low Carbon Steel. They are applicable for all grades of low carbon steel up to a yield point of 70,000 lb. per sq. in. and are based on a factor of safety of three. The welding procedure shall conform to that set forth in the Recommended Practice published by the American Welding Society, dated August 1944.)

4.3—WELDS CONNECTING TWO CHANNELS TO FORM AN I-SECTION FOR USE AS A BEAM

The required tension strength of welds connecting two channels to form an I-beam shall be determined from the following formula:

$$S_w = \frac{mqs}{2c} \quad \text{where}$$

S_w = required strength of weld in tension, (pounds)

s = longitudinal spacing of welds, (inches)

c = vertical distance between the two rows of welds near or at top and bottom flange, (inches)

q = intensity of load per linear inch of beam, (pounds)
(For method of determination, see below.)

m (for simple channels without stiffening lips at the outer edges)

$$= \frac{w^2}{2w + h/3}, \text{ the distance of shear center of channel from axis of web.}$$

m (for C-shaped channels with stiffening lips at the outer edges)

$$= \left(\frac{wh}{2}\right)\left[\frac{wh + 2d(h - d)}{wh^2 + h^3/6 + d(h - d)^2}\right], \text{ the distance of shear center of}$$

channel from axis of web.

w = projection of flanges beyond web, in inches. (For channels with flanges of unequal width, w shall be taken as the width of the wider flange.)

h = depth of channel or beam, (inches)

d = depth of lip, (inches)

See Examples in Appendix 3.

The intensity of load, q, is obtained by dividing the magnitude of concentrated loads or reactions (in pounds) by the length of bearing or by longitudinal spacing of welds, s, (in inches), whichever is larger. For beams designed for "uniformly distributed load," the intensity q shall be taken equal to three times the intensity of the uniformly distributed design load (in pounds per linear inch).

The required strength of welds depends upon the intensity of the load directly at the weld. Therefore, if uniform diameter and spacing of welds are used over the whole length of the beam, the necessary strength of the welds shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing either one of the following methods may be adopted: (a) the weld spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The strength *in shear* of the welds connecting these plates to the flanges shall then be determined from the formula for S_w specified herein but "c" shall then represent the depth of the beam.

SECTION 5. DESIGN OF BRACED WALL STUDS

The safe load-carrying capacity of a stud may be computed on the basis that wall material or sheathing (attached to the stud) furnishes adequate lateral support to the stud in the plane of the wall, provided the wall material and its attachments to the stud comply with the following requirements:

(a) Wall material or sheathing must be attached to both faces or flanges of the stud.

(b) The spacing of attachments of wall material to stud shall not exceed "a" as determined from the formula

$$a = r_2 \sqrt{\frac{29{,}500{,}000}{f_y}}$$

where $r_2 = \sqrt{I_2/A}$ the radius of gyration of the stud corresponding to the moment of inertia I_2 about the principal axis of the stud perpendicular to the wall, in inches.

f_y = minimum specified yield point of steel, lb. per sq. in.

(c) The modulus of elastic support, k, to be exerted laterally by the wall material and its attachments to the stud, shall be not less than —

for Steel of Grade A, $k = \frac{4.5\ a\ A^2}{I_2}$

for Steel of Grade B, $k = \frac{3.7\ a\ A^2}{I_2}$

for Steel of Grade C, $k = \frac{2.6\ a\ A^2}{I_2}$

for steel of grade other than Grades A, B, and C:

$$k = \frac{f_y^2\ a\ A^2}{240{,}000{,}000\ I_2}$$

where

f_y = yield point of the steel in the studs, pounds per square inch.

a = spacing of attachments of wall material to stud measured along the length of stud, ($a = 1$ for continuous attachment), in inches.

A = area of cross section of stud, in square inches.

I_2 = moment of inertia of cross section of stud about its principal axis perpendicular to wall, inches^4.

k = spring constant or modulus of elastic support of wall material (on each [one] side of stud) plus attachment, i.e., $k = F/y$ where F is the force in pounds which produces an elongation of y inches of a strip of wall material of width "a" and of length equal to the distance between adjacent studs.

See Examples in Appendix 3.

Whether or not a given wall material satisfies this "k" requirement shall be established by test procedure as defined in Appendix 4. Also included in Appendix 4 are a number of "k" values as determined by the test procedure described, for several common types of wall sheathing.

(d) The lateral force, F, which each single attachment of the wall material shall be capable of exerting on the stud in the plane of the wall (in order to prevent lateral buckling of the stud) shall not be less than

$$F_{min} = \frac{k_{min} e P}{2\sqrt{EI_2 k_{min}/a} - P}$$

where

k_{min} = modulus of elastic support, value to be determined from paragraph (c) of this section.

$e = \dfrac{\text{stud length in inches}}{240}$

P = design load on stud, in pounds.

I_2 = *moment of inertia of stud about its principal axis perpendicular* to the wall, in inches^4.

a = spacing of attachments measured along stud, in inches, (a = 1 inch for continuous attachment).

E = Modulus of Elasticity = 29,500,000 lb. per sq. in.

See Examples in Appendix 3.

Whether or not a given means of attachment satisfies this requirement shall be ascertained by test procedure as specified in Appendix 4.

SECTION 6. TESTS

6.1—TESTS FOR SPECIAL CASES

Where elements, assemblies, or details of structural members formed from sheet or strip steel are such that calculation of their strength, safe load-carrying capacity, deflection, or other properties cannot be made in accordance with provisions of this Specification, such properties may be determined by suitable tests.

6.2—TEST PROCEDURE

It is recommended that tests for the purposes defined in Section 6.1 be conducted in accordance with the following procedure.

(a) Where practicable, evaluation of test results should be made on the basis of the mean values resulting from tests of not less than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all tests does not exceed ± 10%. If such deviation from the mean exceeds 10%, at least three more tests of the same kind should be made. The average of the three lowest values of all tests made should then be regarded as the result of the series of tests.

(b) Determinations of allowable load-carrying capacity should be made on the basis that the member, assembly or connection should be capable of sustaining a total load of twice the design load, and that harmful local distortions should not develop at a total load equal to the design dead load plus one and one-half times the design live load.

(c) Determination of elastic properties should be based on the deformations developed at either (1) 75% of the maximum load which can be sustained or (2) a total load equal to the design dead load plus one and one-half times the design live load, whichever is the larger.

(d) Tests shall be made by an independent testing laboratory or by a manufacturer's testing laboratory.

APPENDIX 1

ILLUSTRATIVE SKETCHES

This Appendix contains, on Charts 1 and 2, a number of sketches illustrating what is meant by the actual flat width, w, and the effective design width, b, for various types of cross-sections having stiffened compression elements.

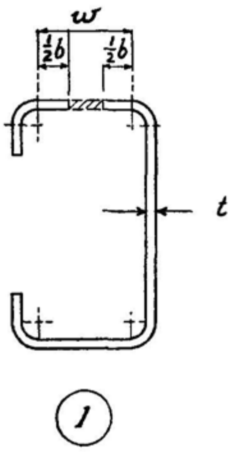
Figures 1 to 5 of Chart 1 show cross-sections of flexural members (top flange in compression). The effective parts of the cross-sections are shown in full lines. The parts which are considered removed for design purposes (see Section 2.3.1) are shown in dotted lines (cross-hatched). Cross-sectional areas, moments of inertia, section moduli, etc., shall be computed for the effective (full line) part of the cross-section only.

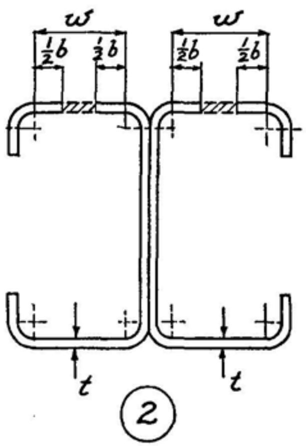
Figures 6 to 8 of Chart 2 show cross-sections of compression members. Those portions which are considered removed in computing the factor "Q" (see Section 3.6.1) are shown cross-hatched. However, all properties, cross-sectional areas, moments of inertia, etc., are to be computed for the full unreduced section.

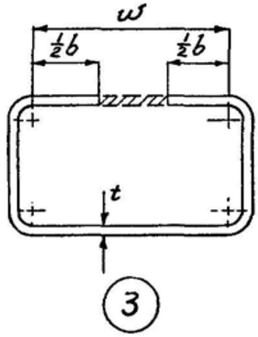
Only cross-sections composed of flanges or webs stiffened along both longitudinal edges are shown in Charts 1 and 2. If sections are composed in whole or in part of flanges stiffened along one edge only, such flanges shall be considered as fully effective, but if the value of w/t exceeds 12, the member shall be designed at reduced unit stresses in accordance with the stipulations of Section 3.2 of this Specification.

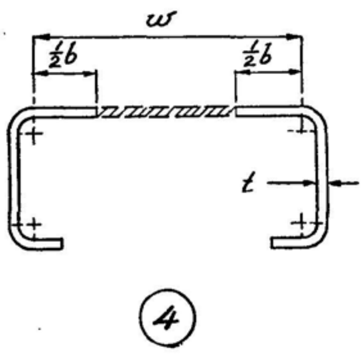
EFFECTIVE CROSS SECTIONS
OF MEMBERS IN BENDING

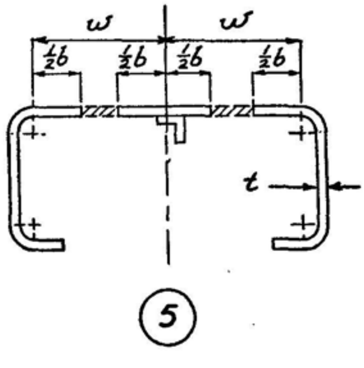
CHART No. 1









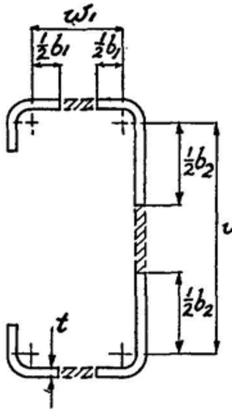


EFFECTIVE DESIGN AREA FOR DETERMINING "Q" FOR CROSS SECTIONS OF MEMBERS IN COMPRESSION

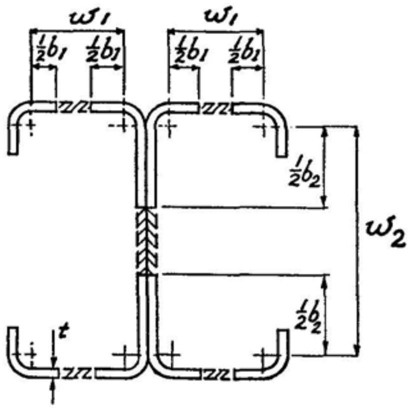
CHART No. 2

(See Section 3.6.1)

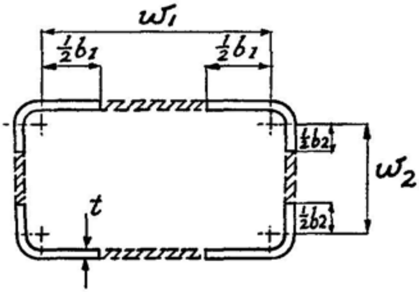
$$Q = \frac{\text{Total Area} - \text{Shaded Area}}{\text{Total Area}} \left(\begin{array}{l}\text{Where All Elements Are}\\ \text{Stiffened At Both Edges}\end{array}\right)$$



6



7



8

APPENDIX 2

DESIGN CHARTS

This Appendix contains two design charts, Nos. 3 and 4, which may be used in applying the provisions of Section 2.3.1 covering effective design width, and Section 3.6.1 covering compression members.

Chart No. 3 is a graph of the formulae of Section 2.3.1 giving effective width for various flat-width ratios. It may be used for any grade of steel at any of the plotted or interpolated stresses, for both load and deflection determinations.

Briefly, the explanation of the two sets of stress values shown in Chart No. 3 lies in the fact that the effective design width varies inversely with the unit stress, becoming less as the unit stress increases. Consequently, in order to develop a yield-strength safety factor of 1.85 (corresponding to that specified for basic working stresses in Section 3.1) the effective width for load determinations must be that corresponding to a load 1.85 times the actual safe load. The effective width, "b", for safe load determinations, therefore, is not that corresponding to the actual stress under the safe load, but to the unit stress under 1.85 times that safe load. Hence, the "b" values, based upon the curve values shown under "Use for Load Determinations," are those corresponding to unit stresses 1.85 times those designated.

In determining deflection, the "b" value corresponding to the *actual* unit stress under the design load must be used, however, for reasonably accurate results. That "b" value (corresponding to the actual unit stress) is greater than the "b" value used for allowable load determinations, since the latter corresponds to the unit stress under 1.85 times the actual design load, as explained in the preceding paragraph.

Thus two different values appear on each curve of Chart No. 3 — one for "load determination" giving "b" values for adjusted unit stresses (1.85 times those designated), the other for "deflection determination" giving "b" values corresponding to the actual unit stress under the load for which deflection is being computed.

In unsymmetrical sections subjected to bending, having a top flange much wider than the bottom flange (Examples 3 and 4 of Appendix 3) little difference would result, generally, were load capacity calculated from the "deflection" values; because in such sections, although a decreasing "b" causes "I" to decrease with increasing unit stress, it also causes the critical distance from the neutral axis to the bottom fiber to decrease, thus resulting in little net change in the section modulus.

In the case of symmetrical or nearly symmetrical sections, however, use of the "deflection" values (based on the actual unit stress) for load determinations would result in an appreciably lower factor of safety than 1.85, because in such members, as the load and unit stress are increased, not only does "I" decrease with "b"; but, in addition, there is an increase in the distance from neutral axis to top fiber (which governs in symmetrical members because of the reduced effective width of the top flange) thus making even greater reduction in the section modulus. The situation is similar, but more pronounced, in the case of members under axial compression.

The column curve, Chart No. 4, shows, for Grade C steel, column design curves based on the formulae of Section 3.6.1 for various Q values up to 1.00. For other grades of steel the formulae of Section 3.6.1 may be used directly or similar curves constructed.

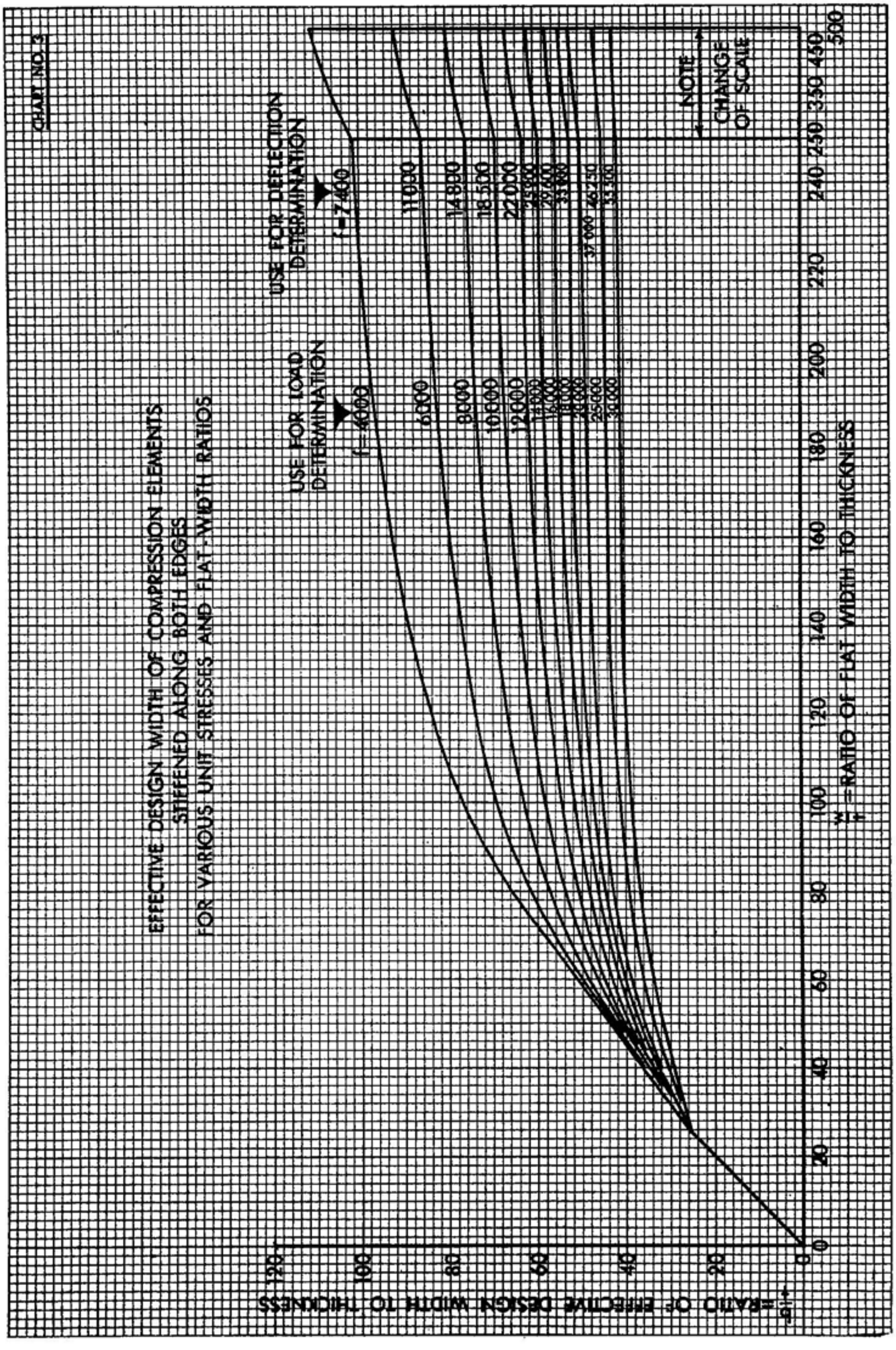


CHART NO. 3
EFFECTIVE DESIGN WIDTH OF COMPRESSION ELEMENTS
STIFFENED ALONG BOTH EDGES
FOR VARIOUS UNIT STRESSES AND FLAT-WIDTH RATIOS
USE FOR LOAD DETERMINATION
f=4000
6000
8000
10000
12000
25000
30000
USE FOR DEFLECTION DETERMINATION
f=7400
11000
14800
18500
22000
37000
46250
55500
NOTE
CHANGE OF SCALE
w/t =RATIO OF FLAT WIDTH TO THICKNESS
b/t =RATIO OF EFFECTIVE DESIGN WIDTH TO THICKNESS

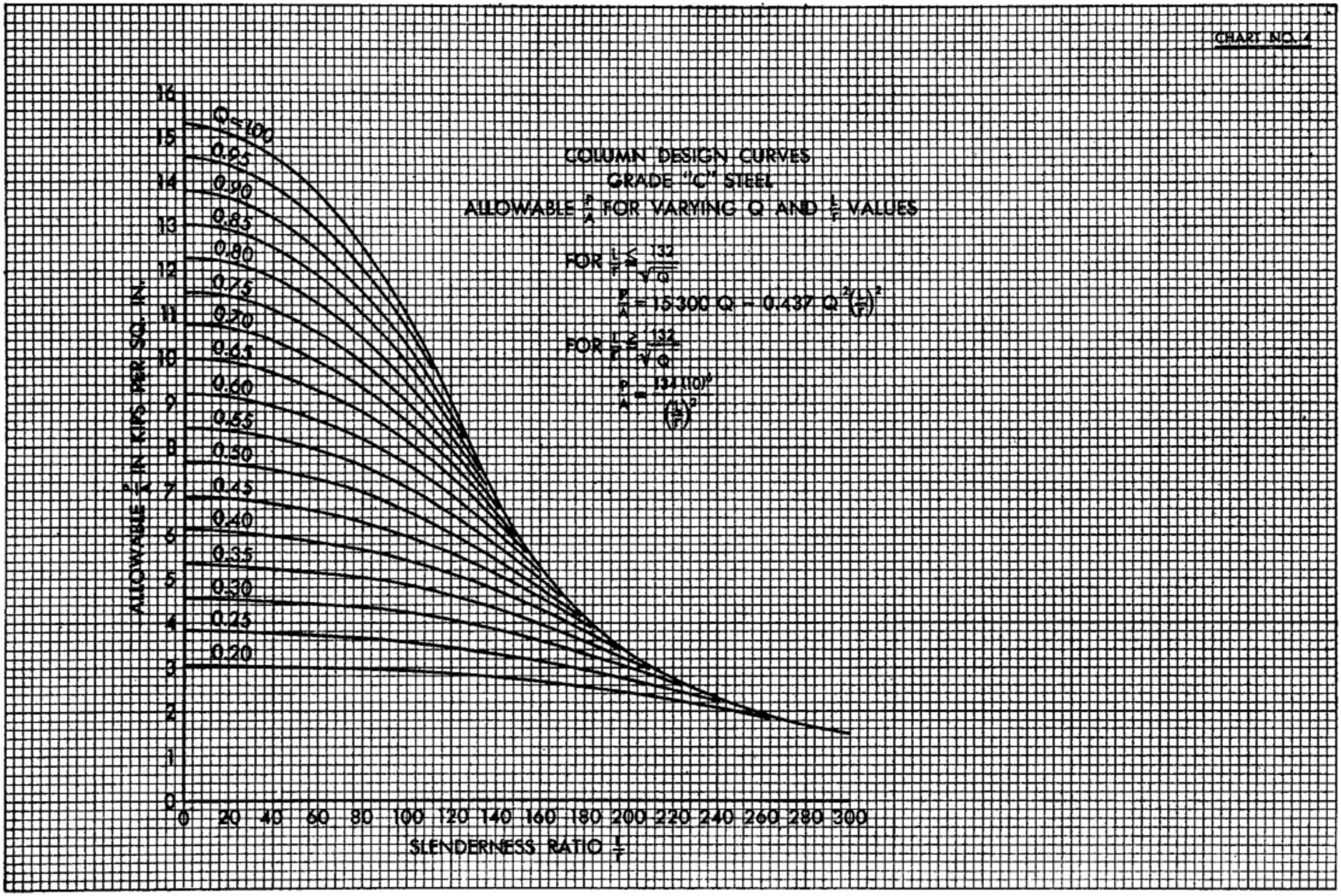


CHART NO. 4
COLUMN DESIGN CURVES
GRADE "C" STEEL
ALLOWABLE $\frac{P}{A}$ FOR VARYING Q AND $\frac{L}{r}$ VALUES
FOR $\frac{L}{r} \leq \frac{132}{\sqrt{Q}}$
$\frac{P}{A} = 15300\,Q - 0.437\,Q^2\left(\frac{L}{r}\right)^2$
FOR $\frac{L}{r} \geq \frac{132}{\sqrt{Q}}$
$\frac{P}{A} = \frac{134(10)^6}{\left(\frac{L}{r}\right)^2}$
Q=1.00
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
ALLOWABLE $\frac{P}{A}$ IN KIPS PER SQ. IN.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
SLENDERNESS RATIO $\frac{L}{r}$

APPENDIX 3

DESIGN EXAMPLES

The examples calculated in detail in this Appendix have been selected to illustrate those provisions of the Specification which pertain to the strength and stability of thin compression flanges and webs. The examples are intended to illustrate those features which differ from standard structural design procedure.

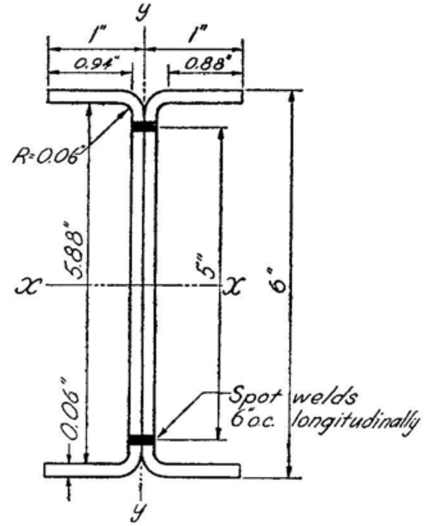
The variety of shapes which it is possible to fabricate of sheet or strip steel is practically unlimited. In order to show the application of the Specification to various types of cross-sections, a number of different shapes are investigated. The same principles may be applied to most other possible rectilinear cross-sections.

In the examples which follow all of the sections have small-radius bends at the corners. In some of those examples (Nos. 3, 4 and 5) the properties have been computed on the basis of square corners for the sake of simplicity. More accurate results will be obtained if the properties are based on the actual contour, but the difference will usually not exceed a few per cent.

DESIGN EXAMPLES

Members In Bending

EXAMPLE No. 1



Steel: Grade "C"
Span L = 12 feet

To find: Allowable uniformly distributed load.

(a) Top Flange is Braced Laterally all along the Span.

The compression flanges of the beam are stiffened against buckling only along the joints with the web, and are not stiffened at the outer edges. Consequently, the full section is to be used in design, and the allowable unit stress determined in accordance with Section 3.2 of the Specification.

Properties: $I_x = 4.07$ in.^{4} $\quad r_y = 0.294$ in.
$S_x = 1.36$ in.^{3}

Allowable stress from Section 3.2 (b):

$$\frac{w}{t} = \frac{0.88}{0.06} = 14.67$$

$$f_c = [1.67\ (18000) - 5430] - \frac{(18000 - 8150)}{18}\ (14.67) = 16{,}540 \text{ p.s.i.}$$

$$M = (16540)\ (1.36) = 22490 \text{ in. lb.}$$

Allowable uniform load: $M = \dfrac{WL}{8}$, whence

$$W = \frac{22490(8)}{12(12)} = \underline{1248 \text{ lb.}} = \underline{104 \text{ lb./ft.}}$$

From Section 3.5 (d) the minimum bearing to prevent web crippling:

$$\text{End Reaction} = \frac{104(12)}{2} = 624 \text{ lb.}$$

$$B_{min} = 0.06 \left[\frac{624}{0.93(0.06)^2(18000)} - 8.00 \right]^2 = 0.333 \text{ in.}$$

Use $B = \underline{2 \text{ inches}}$

Welds connecting the two channels (Section 4.3):

$$m = \frac{(0.94)^2}{2(0.94) + 6/3} = 0.228 \text{ in.}$$

$$c = 5.0 \text{ in.}$$

At ends, $R = qs = 624$ lb.

$$S_w = \frac{624 \times 0.228}{2 \times 5} = \underline{14.2 \text{ lb.}}$$

Under uniformly distributed load,

$$q = \frac{3 \times 104}{12} = 26 \text{ lb./in.}$$

$$s = 6 \text{ in.}$$

$$S_w = \frac{26 \times 6 \times 0.228}{10} = \underline{3.6 \text{ lb.}}$$

Since the design strength per spot as listed in Table 4.2.2 for $t = 0.06$ inches is 725 lb., a longitudinal spacing of 6 inches provides more than sufficient strength.

Web Shear (Section 3.4)

$$\text{Allowable } v = \frac{64{,}000{,}000}{(h/t)^2}$$

$h = 5.88$ in., $t = 0.06$ in., $h/t = 98$

$$v = \frac{64{,}000{,}000}{(98)^2} = 6660 \text{ lb./sq. in.}$$

Total Allowable Shear $= 6660 \times 0.06 \times 5.88 \times 2 = 4700$ pounds

The actual shear $= 104(6) = 624$ lb.

The webs are therefore adequate.

(b) Same Beam; but Unbraced Length of Compression Flange = 6 feet.

Allowable stress according to Section 3.3:

$L/r_y = 72/0.294 = 245$

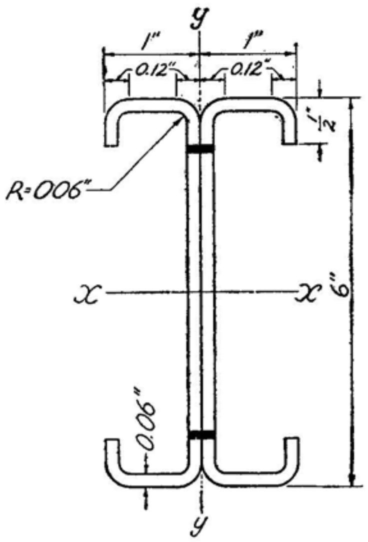
$f'_c = 250{,}000{,}000/(245)^2 = 4170$ p.s.i.

The allowable stress according to Section 3.2 is 16540 p.s.i. (Unstiffened Compression Elements) and 4170 p.s.i. according to Section 3.3 (Laterally Unbraced Compression Flanges). The smaller of the two governs. Then

$M = 4170\,(1.36) = 5670$ in. lb. and

$$W = \frac{5670(8)}{12(12)} = \underline{312 \text{ lb.}} \text{ or } 26 \text{ lb./ft.}$$

EXAMPLE No. 2



Steel: Grade "C"

Span L = 12 feet. Continuous lateral bracing along top flanges.

To find: Allowable uniformly distributed load.

This beam is identical with that of Example No. 1 except that stiffening lips have been added at the outer edges of the flanges.

Checking requirements for lip dimensions (Sections 2.2 and 2.3.2.1):

$$\frac{w}{t} = \frac{1 - 2(0.12)}{0.06} = 12.67, \text{ whence } \frac{d}{t} = 2.8\sqrt[6]{(12.67)^2 - 144} = 4.47 \text{ and}$$

$$d = 4.47\,(0.06) = 0.27 \text{ in. min. out to out depth.}$$

The ½ inch lips satisfy this requirement.

The compression flanges of this beam are adequately stiffened along both longitudinal edges. Consequently, they are to be designed according to the provisions of Section 2.3.1.

From Section 2.3.1, flanges with w/t less than 25 are fully effective (b/t equals w/t).

Properties: $I_x = 4.81$ in.^{4}
$S_x = 1.60$ in.^{3}

Allowable stress $f_b = 18000$ p.s.i. from Section 3.1

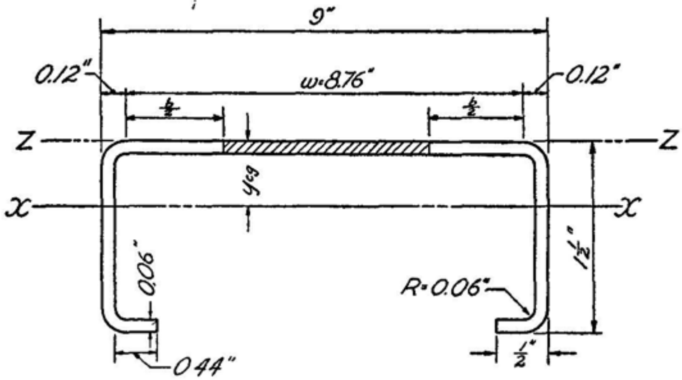
$$M = (18000)\ (1.60) = 28800 \text{ in. lb.}$$

$$W = \frac{28800(8)}{12(12)} = \underline{1596 \text{ lb.}} \text{ or } \underline{133 \text{ lb./ft.}}$$

A comparison of this load with that of Example 1 (a) shows that a gain of allowable load of 28% is achieved by adding lips to the beam of Example 1 (a). These stiffening lips required only about 11% additional material. This example illustrates the advantages which can be gained by using favorably shaped sections.

EXAMPLE No. 3

Steel: Grade "C"
To find: Resisting moment, *When Stress Governs.*



For an Approximation Use an Effective Width of the Top Flange Based on f = 18000 p.s.i.

$$\frac{w}{t} = \frac{8.76}{0.06} = 146$$

From Chart 3 of Appendix 2, the effective design width, b, exclusive of corners = 50.0t = 3.00 in.

Total effective width = 3.00 + 2(0.06) = 3.12 in. web to web.

Cross-sectional properties (assuming square corners):

Element	Area = F (in.^{2})	y (in.) (Dist. from top fiber)	(F)y (in.^{3})	(F)y^2 (in.^{4})
Top Flange	3.12 × 0.06 = 0.1872	0.03	0.0056	0.0002
Webs	2 × 1.50 × 0.06 = 0.1800	0.75	0.1350	0.1012
Bottom Flanges	2 × 0.44 × 0.06 = 0.0528	1.47	0.0776	0.1141
Summation	0.4200		0.2182	0.2155

$$+\ I_{cg}\text{(of webs)}\ ^* = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$0.2493 = I_z$$

$$-\ 0.4200(0.519)^2 = 0.1131$$

$$\text{Moment of Inertia: } I_x = 0.1362$$

* Moments of Inertia of top and bottom flanges about their own centroidal axes have been neglected.

Distance of axis from top fiber: $y_{cg} = \frac{0.2182}{0.4200} = 0.519$ in.

Section Modulus: $S = \frac{0.1362}{1.500 - 0.519} = \frac{0.1362}{0.981} = 0.139 \text{ in.}^3$

Resisting Moment: $M = 18{,}000(0.139) = 2500$ in. lb.

Due to the eccentric position of the neutral axis the compressive stress in this section is considerably smaller than the tensile stress. When the tensile stress in the bottom flange is equal to the working stress, 18000 p.s.i., the corresponding compressive stress in the top flange is easily found if the position of the neutral axis is known. This position, however, depends on the effective width of the top flange, which in turn depends on the stress in that flange. It is therefore desirable to locate the neutral axis accurately by successive approximations.

2nd Approximation:

Guided by the determination in the first approximation:

Assume $y = 0.45$ in.

Then the compressive stress in the top flange:

$$f_c = \frac{18000(0.45)}{1.05} = 7710 \text{ p.s.i.}$$

From Chart 3 in Appendix 2, for this f_c and $w/t = 146$:

$b/t = 71.3$, thus the effective design width, $b = 71.3\,(0.06) = 4.28$ in.

Total effective width $= 4.28 + 2\,(0.06) = 4.40$ in.

Check assumed position of the neutral axis (assuming square corners):

Element	Area = F (in.^{2})	y (in.) (Dist. from top fiber)	(F)y (in.^{3})
Top Flange	$4.40 \times 0.06 = 0.2640$	0.03	0.0079
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350
Bottom Flanges	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776
Summation	0.4968		0.2205

Distance of axis from top fiber: $y_{cg} = \frac{0.2205}{0.4968} = 0.443$ in.

3rd Approximation:

On the basis of the determination above assume $y = 0.443$ in.

$$f_c = \frac{18000(0.443)}{1.057} = 7545 \text{ p.s.i.}$$

From Chart 3 in Appendix 2, for this f_c and $w/t = 146$:

$b/t = 71.5$, thus the effective design width, $b = 71.5\ (0.006) = 4.29$ in.

Total effective width $= 4.29 + 2\,(0.06) = 4.41$ in.

Cross-sectional properties (assuming square corners):

Element	Area = F (in.^{2})	y (in.) (Dist. from top fiber)	(F)y (in.^{3})	(F)y^2 (in.^{4})
Top Flange	$4.41 \times 0.06 = 0.2646$	0.03	0.0079	0.0002
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350	0.1012
Bottom Flanges	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776	0.1141
Summation	0.4974		0.2205	0.2155

$$+ I_{cg}(\text{of webs})^* = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$\overline{0.2493} = I_x$$

$$- 0.4974(0.443)^2 = 0.0976$$

$$\text{Moment of Inertia: } I_x = 0.1517$$

Distance of axis from top fiber:

$$y_{cg} = \frac{0.2205}{0.4974} = 0.443 \text{ in. (checks assumed value)}$$

$$\text{Section Modulus: } S = \frac{0.1517}{1.500 - 0.443} = \frac{0.1517}{1.057} = 0.144 \text{ in.}^3$$

Resisting Moment: $M = 18{,}000(0.144) = \underline{2590 \text{ in. lb.}}$

A comparison of these values with the results of the first approximation shows that the final accurate determination allows an increase in design load of about 3.6% (if stress governs) as compared with the values obtained in the simpler, but approximate, determination based on a stress of 18000 lb. per sq. in. in the compression flange.

EXAMPLE No. 4

Steel: Grade "C"

To find: Moment of Inertia for *Computing Deflection*

Same Section as Example No. 3, loaded to its capacity Resisting Moment.

Since the top flange stress, from Example No. 3, is 7545 p.s.i., that stress will be used for a first approximate computation of the effective width of top flange to be used for deflection determinations.

$w/t = 146$ (as in Example No. 3)

From Chart 3 of Appendix 2, the effective design width, b, exclusive of corners $= 89.6t = 5.38$ in.

Total effective width $= 5.38 + 2(0.06) = 5.50$ in. web to web.

Properties, assuming square corners, are:

Moment of Inertia: $I = 0.161$ in.^{4}

Distance of neutral axis from top fiber: $y = 0.396$ in.

* Moments of Inertia of top and bottom flanges about their own centroidal axes have been neglected.

Stress in top flange: $f_c = \frac{2590(0.396)}{0.161} = 6370$ p.s.i.

2nd Approximation:

Guided by the determinations of the first approximation, now assume that the compressive stress, f_c, in the top flange is 5900 p.s.i.

Since this value of f is beyond the limits of Chart 3 in Appendix 2, b/t is determined from the formula of Section 2.3.1:

$$\frac{b}{t} = \frac{10{,}320}{\sqrt{5900}}\left(1 - \frac{3120}{146\sqrt{5900}}\right) = 97.0,$$

whence the effective design width, $b = 97.0\ (0.06) = 5.82$ in.

Total effective width $= 5.82 + 2(0.06) = 5.94$ in.

Properties, assuming square corners, are:

Moment of Inertia: $I_x = 0.1648$ in.^{4}

Distance of neutral axis from top fiber: $y = 0.379$ in.

Stress in top flange: $f_c = \frac{2590\ (0.379)}{0.1648} = 5960$ p.s.i.

3rd Approximation:

On the basis of the determination above, assume $f_c = 5980$ p.s.i.

Since this value of f is beyond the limits of Chart 3 in Appendix 2, b/t is again determined from the formula of Section 2.3.1:

$$\frac{b}{t} = \frac{10{,}320}{\sqrt{5980}}\left(1 - \frac{3120}{146\sqrt{5980}}\right) = 96.6,$$

whence the effective design width, $b, = 96.6(0.06) = 5.80$ in.

Total effective width $= 5.80 + 2(0.06) = 5.92$ in.

Cross-sectional properties (assuming square corners):

Element	Area = F (in.)^2	y (in.) (Dist. from top fiber)	(F)y (in.^{3})	(F)y^2 (in.^{4})
Top Flange	$5.92 \times 0.06 = 0.3552$	0.03	0.0107	0.0003
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350	0.1012
Bottom Flanges	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776	0.1141
Summation	0.5880		0.2233	0.2156

$$+ I_{og}\text{ (of webs)*} = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$0.2494 = I_x$$

* Moments of Inertia of top and bottom flanges about their own centroidal axes have been neglected.

Distance of neutral axis from top fiber:

$$y_{cg} = \frac{0.2233}{0.5880} = 0.380 \text{ in.}$$

$-\ 0.5880(0.380)^2 =$	$-\ \underline{0.0848}$
Moment of Inertia: I_x	$=\ 0.1646$

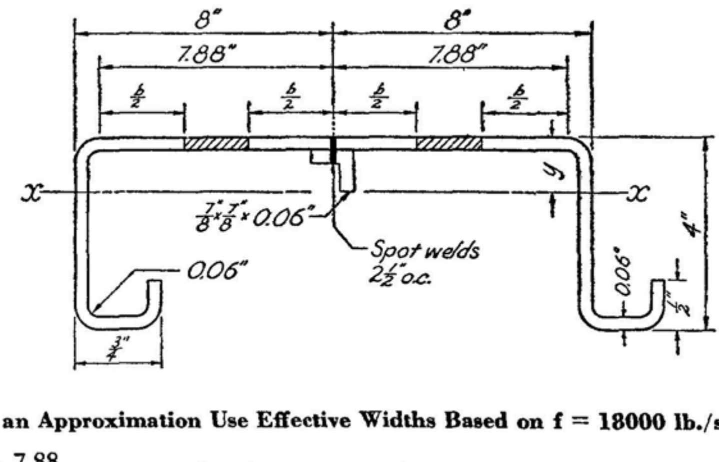
$$\text{Stress in top flange} = \frac{2590(0.380)}{0.1646} = 5980 \text{ p.s.i. (checks assumed stress)}$$

A comparison of the value of 0.1646 in.^{4} for the moment of inertia, I, (for deflection) with the value of 0.1517 in.^{4} which was used in the stress calculations of Example 3 shows that the deflection, as determined, will actually be approximately 8% less than if computed on the basis of the same moment of inertia used in the stress calculations. It will also be noted that the first approximation of the deflection determinations gave a value of I which was within approximately 2% of the correct value. See discussion of Design Chart No. 3 in Appendix No. 2 on Page 23.

EXAMPLE No. 5

Steel: Grade "C"

To find: Resisting moment and effective moment of inertia.



(a) For an Approximation Use Effective Widths Based on f = 18000 lb./sq. in.

$$\frac{w}{t} = \frac{7.88}{0.06} = 131; \ I_0 \text{ of stiffener} = 0.007 \text{ in.}^4$$

From Chart 3 in Appendix 2: Effective design width, b, exclusive of corners ($f = 18000$) $= 49.4t = 2.96$ in. Total effective width $= 2.96 + 0.06 = 3.02$ in.

For intermediate stiffeners the required moment of inertia is twice that required for an edge stiffener (Section 2.3.2.1) thus

$$I_{min} = 2(1.83)(0.06)^3(7.88) = 0.00624 \text{ in.}^4$$

The stiffener shown satisfies this requirement.

Cross-sectional properties, assuming square corners, and using an effective width, b, of top flange based upon a stress of 18000 lb. per sq. in. (for an approximate solution), are:

Moment of Inertia: $I = 2.473$ in.^{4}
Section Modulus: $S = 0.955$ in.^{3}
Resisting Moment: $M = 18000(0.955) = \underline{17190 \text{ in. lb.}}$

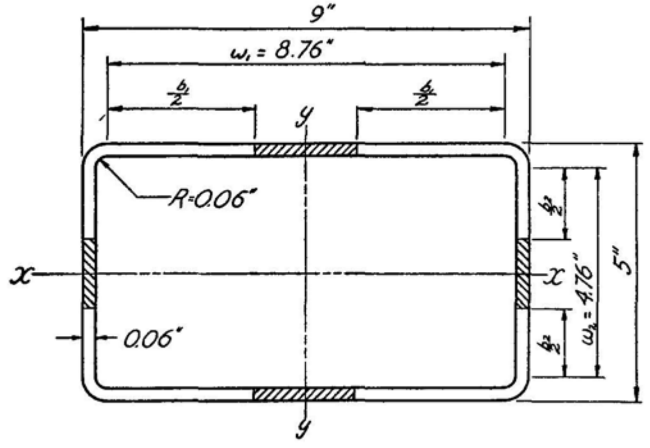
Section 2.3.2.2 requires that the intermediate stiffener be connected to the compression flange at intervals not to exceed $\frac{w}{3} = \frac{7.88}{3} = 2.63$ inches. The 2½" weld spacing shown on the sketch is adequate.

(b) A more accurate solution may be had by locating the exact position of the neutral axis by successive approximations and using the exact compressive stress in the top flange. This procedure is outlined in Example 3.

Members in Axial Compression

EXAMPLE No. 6

Steel: Grade "C"
Length $L = 15$ feet.
To find: Allowable axial load, P (Section 3.6)



In accordance with Section 3.6 the sectional properties are to be computed on the basis of full section, thus:

$A = 1.66$ in.^{2}
$r_x = 2.16$ in.

The value of Q (Section 3.6.1) is determined as follows:

$$\frac{w_1}{t} = \frac{8.76}{0.06} = 146; \quad \frac{w_2}{t} = \frac{4.76}{0.06} = 79.3$$

Effective design widths for $f = 18000$:

$b_1 = 50.0t = 3.00$ in.; $b_2 = 44.5t = 2.67$ in. thus the Effective Area:

$$A_{eff} = 1.66 - 0.06\,[2(8.76 - 3.00) + 2(4.76 - 2.67)] = 0.72 \text{ in.}^2$$

Since member is completely stiffened:

$$Q_A = \frac{\text{Effective Area}}{\text{Gross Area}} = \frac{0.72}{1.66} = 0.43$$

$$\text{Slenderness Ratio, } \frac{L}{r_x} = \frac{15 \times 12}{2.16} = 83.3$$

From Column Design Curve in Chart 4 of Appendix 2, for $Q = 0.43$, $L/r_x = 83.3$,

$P/A = 6020$ p.s.i. acting on full area of section.

$P = 6020(1.66) = \underline{9990 \text{ lb.}}$

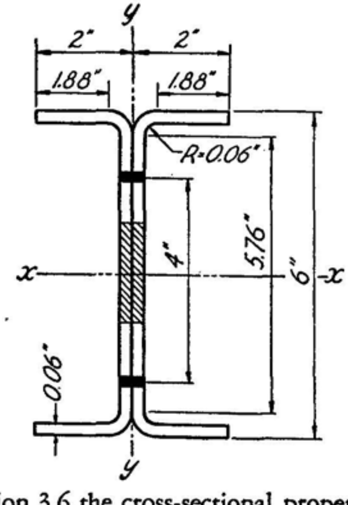
EXAMPLE No. 7

Steel: Grade "C"

Length $L = 15$ feet

Stud faced on both flanges by wall-sheathing of adequate strength.

To find: Allowable design load and maximum spacing of wall-sheathing attachments.



In accordance with Section 3.6 the cross-sectional properties are to be computed on the basis of the full section, whence

$A = 1.176$ in.^{2}	
$I_x = 6.18$ in.^{4}	$I_y = 0.641$ in.^{4}
$r_x = 2.29$ in.	$r_y = 0.74$ in.

The value of Q (Section 3.6.1) is determined as follows:

The member is composed of both stiffened and unstiffened elements. Paragraph (c) of Section 3.6.1 therefore applies.

$$\text{For the flanges, } w/t = \frac{1.88}{0.06} = 31.3$$

Since the flanges are unstiffened elements,
from Section 3.2: $f_c = 12600 - 148.5(31.3) = 7950$ lb. per sq. in.,

whence $$Q_s = \frac{7950}{18000} = 0.44$$

For the webs (stiffened elements):

$$w/t = \frac{5.76}{0.06} = 96^*$$

b/t (Section 2.3.1 and Chart No. 3), based on f = 7950 lb. per sq. in., equals 62. Since the full area of all unstiffened elements (as well as the effective [reduced] area of all stiffened elements) is to be included in computing Q_a—according to Section 3.6.1 Par. (c)—the reduction of web (stiffened) section is first calculated:

$$w - b = (96 - 62)\ t = (34)\ (0.06) = 2.04 \text{ in.}$$

The corresponding area reduction (for the two webs) is:

$$2.04 \times 0.06 \times 2 = 0.245 \text{ in.}^2$$

The net area of the stud is:

$$1.176 - 0.245 = 0.931 \text{ in.}^2$$

Thus, $Q_a = \dfrac{0.931}{1.176} = 0.79$

Then $Q = Q_s \times Q_a = (0.44)(0.79) = 0.35$

Maximum allowable spacing of wall-sheathing attachments from Section 5:

$$a_{max} = r_y \sqrt{\frac{29{,}500{,}000}{33{,}000}} = 30r_y = 30(0.74) = 22.2 \text{ in.}$$

Chosen spacing: a = 22.0 in. to afford lateral bracing in plane of wall.

Slenderness ratio, (perpendicular to wall): $\dfrac{L}{r_x} = \dfrac{15(12)}{2.29} = 78.6$

From the column design curves in Chart 4 of Appendix 2, for

$$Q = 0.35 \text{ and } \frac{L}{r} = 78.6$$

$P/A = 5050$ p.s.i. acting on full area of section

$P = 5050\ (1.176) = \underline{5940 \text{ lb.}}$

To determine the properties of the wall materials and attachments required to adequately brace the studs laterally, refer again to Section 5. From paragraph (c) of that section,

$$k = \frac{2.6\ a\ A^2}{I_2} = \frac{2.6 \times 22 \times \overline{1.176}^2}{0.641}$$

$$= \underline{123}$$

From paragraph (d) of Section 5,

$$F_{min} = \frac{k\ e\ P}{2\sqrt{E\ I_2\ k/a} - P}$$

$$= \frac{123 \times 0.75 \times 5940}{2\sqrt{29{,}500{,}000 \times 0.641 \times 123/22} - 5940}$$

$$= \underline{37.5 \text{ lb.}}$$

* If the weld spacing is such that the two web elements may be considered acting as one, the w/t value may be taken as 48.

APPENDIX 4

TESTING OF WALL MATERIALS FOR LATERAL BRACING VALUE

Where wall material is utilized as lateral bracing of wall stud members in accordance with Section 5 of this Specification, its properties may be established by the following testing procedure:

Specimens — Test specimens shall be made up from two strips of the wall material, the width of each being equal to twice the spacing (along the stud) of the attachments which are to be used. Where continuous attachment of the wall sheathing to the studs is contemplated, the width of specimen shall be not less than 6 inches.

Two short pieces of the studs (of the type to be used) shall be attached to the wall material at a distance equal to the usual stud spacing. The attachments shall be located symmetrically with respect to the center lines of the specimen of wall material. In order to simulate conditions under which wall board is continuous and is attached to studs without joints, the wall board shall protrude 6 inches beyond the stud on each side. In order to simulate conditions under which wall board is joined to the next board along the stud, such a joint shall be made by attaching to the stud on its free edge a strip of wall board 6 in. wide and by joining it to the board that connects the studs in the manner proposed for use in the structure. The number of specimens to be tested shall be as specified in Section 6 of this Specification.

Testing Procedure — The test specimen assemblies described above shall be tested in tension in a standard testing machine. By means of suitable connections the loads shall be applied concentrically to each assembly, through the studs.

An initial load P_0 of the order of 50 lb. shall be applied to the assembly. The load shall then be increased by uniform increments. At each increment the distances between the attachments in each of the four pairs shall be measured to the nearest thousandth of an inch. For specimens with continuous attachment the distance between the studs proper shall be measured at each increment. This procedure shall be continued until the specimen fails.

Evaluation of tests — The elongations corresponding to each load shall be computed by subtracting the original distance between attachments at the load P_0 from the distance measured at any particular load. The elongations so obtained for the different attachments shall be averaged for any particular load and a curve of the average elongations (in inches) versus applied load (in pounds) shall be drawn. From this curve the elongation y_1 corresponding to the reference load $P_1 = 0.75\ P_{ultimate}$ shall be obtained. The actual modulus of support of one attachment, k, is then determined for individual attachments from

$$k = \frac{P_1 - P_0}{4y_1}$$

and for continuous attachment from

$$k = \frac{P_1 - P_0}{2y_1 b}$$

Where, b = the width of the wall material specimen, in inches, and the other factors are as described above.

The k value so determined shall be equal to or larger than the value of k required in Section 5.

The strength of one attachment, F, shall be determined from the ultimate test load, P_{ult}, for one individual attachment from the formula:

$$F = \frac{P_{ult}}{4}$$

and for continuous attachment from

$$F = \frac{P_{ult}}{2b}$$

where, b = width of continuously bonded wall material in inches.

The F so determined shall be equal to or larger than F_{min} as specified in Section 5.

k Values Determined by Tests

The tabulation below indicates the approximate magnitude of k values of a few common types of wall sheathing materials, tested in accordance with the procedure outlined above using one type of steel stud and two alternate types of attachment.

	Range of k values
1/2" Standard Density Wood and Cane Fiber Insulating Boards	290—603
1/2" Paper Base Insulating Board	915—1460
3/8" Gypsum Board Sheathing	775—1535
3/16" Medium Density Compressed Wood Fiber Board	2010—4560
5/32" High Density Compressed Wood Fiber Board	3960—7560

The above values are indicative only; the k value for any specific construction will depend upon the particular type of wall sheathing material and method of attachment employed, also the type of steel stud to which attachment is made.

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